AN INVESTIGATION OF PRESTRESSED CONCRETE STRUCTURES

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Thesis
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A THESIS PRESENTED TO THE FACULTY
OF RENSSELAER POLYTECHNIC INSTITUTE
IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR DEGREE OF
MASTER OF CIVIL ENGINEERING

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TROY, NEW YORK
MAY, 1940

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FOREWORD

The object of this thesis is three-fold; first, to present a paper in which a correlation has been made of the existing theories and design formulae pertinent to the subject of prestressed concrete; second, to apply these existing theories to the design of a particular structure and third, to compare the structure so designed to a similar structure using conventional reinforced concrete design procedures.

In the accomplishment of these objectives the authors have made no attempt to present original theories in the derivation of the design formulae.

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ACKNOWLEDGEMENT

The authors wish to express their appreciation to Associate Professor Joseph S. Kinney of the Department of Civil Engineering for his guidance in the selection of this subject and for his suggestions which aided the authors in reaching a conclusion.

INTRODUCTION

The development of the theory of prestressing various concrete structures is not entirely new, nor is the application of the theory to actual construction unknown by any means. For certain structures such as circular pressure vessels this type of construction is widely used. For other structural members, especially straight line components, the application is much more limited at the present time. The authors feel that the advantages to be gained by using prestressing in all types of construction will eventually become more widely known, and hence the process will find ever-increasing application.

The objective of prestressing concrete construction is to eliminate concrete tensile stresses under design load conditions. This enables the designer to utilize the higher compressive stresses of modern concrete and the high tensile properties of cold drawn steel wire.

In general, the establishment of prestressing is accomplished by tensioning the steel reinforcement before the load is applied, the stretching force being transmitted to the concrete as a compressive force after the concrete has attained sufficient strength to take the stresses thus applied. In this manner, stresses of opposite sign to those occuring under load are imparted to the structure.

One of the first proposals to use prestressing was made by Jackson in 1888. The method suggested was that of strengthening the structure by tightening the reinforcement to a degree not determinate.

The idea was carried further by Mandl who established a limit to the concrete tensile stress and to the reduction in cracking. In his work he did not, however, consider losses of the initial prestress and hence the counteraction due to stretching the reinforcement was only partially effective. Later work in this field by Dill, Hewett, Freyssinet, and others arrived at a process whereby full prestressing was utilized, all possible losses being considered and cracklessness being guaranteed due to the absence of any tensile stresses in the concrete under load.

The most recent work in prestressing aims at refinements to the early propositions. Various means have been advanced to achieve the desired degree of prestressing and the application of the process has been extended.

In general, two methods of applying the initial prestress are used; pre-stretching and post-stretching. The terms indicate whether the tensioning is carried out before or after hardening of the concrete. With pre-stretching, the concrete remains in the mold until the stretching can be transmitted to the concrete by bond. At the outset the tensile force in the steel is taken at the ends of the mold by special anchorages which are subsequently removed.

Post-stretching is applied when the concrete has hardened. In this case permanent anchorages at the ends of the structure transmit the compression to the concrete, there being no bond between the concrete and the steel.

With pre-stretching, at the release of the stretching force to the concrete, the initial prestress is immediately reduced due to the elastic deformation of the concrete and to shrinkage, which losses are increased gradually by further shrinkage and by plastic flow of the concrete. With post-stretching the losses due to elastic deformation and to the initial shrinkage are eliminated, thus the use of conventional steel is satisfactory.

In the design of prestressed structures the nature of the structure will determine which of the various systems of prestressing can be used most advantageously and which types of materials are best suited to the job. It must be decided whether full or partial prestressing will be used, based on the characteristics imparted by these two methods.

In full prestressing the stretching force is of such magnitude that no tensile stress occurs under working load and thus cracklessness of the concrete is guaranteed. This latter quality is, of course, highly desirable in pressure vessels such as pipes and tanks under pressure. Due to the fact that the concrete can be prevented from cracking the sections of concrete can be treated as a homogenous material and the design is not, therefore, based on the "cracked section" as is the case in the conventional design. This absence of cracks and absence of tensile stresses in the concrete also increases the shear resistance of the concrete. In certain cases it allows the economical use of high strength steel. The strain of the structure at the release of compression to the concrete is greater than under working load,

which necessitates a high early compressive strength and which may cause inconvenience for transporting and handling precast products, since any additional strain has to be avoided.

In cases where absence of cracks is not necessary as in beams, for example, partial prestressing may be more advantageous. In this case a smaller stretching force is required and is applied to only part of the total reinforcement. This has an effect on the economy over full prestressing because of the reduction in fabrication costs and because of the greater ease of handling precast products. By controlling the degree of initial prestress the strain under working load (deflection in the case of beams) can be controlled so as to obtain any degree of deflection between the limits of the large deformations and heavy cracking of the non-prestressed structure and the extremely small deformation and absence of cracks in the fully prestressed structure.

In the design the losses of the initial prestress mentioned previously must be considered in order that the final compressive stress in the concrete prior to loading may be held within the limits desired. These losses are computed and allowance made for them in the particular case at hand by increasing the initial prestress in the steel.

GENERAL THEORIES OF DESIGN

The application of prestressing to particular structures will now be considered. Since prestressing is most advantageously used in pressure vessels, this type of structure will be considered first.

WIRE WOUND PRESTRESSED CONCRETE PRESSURE PIPE

Conventional reinforced concrete pipes have been used in water supply lines for many years within the limitations of internal pressures and external loadings; but with the comparatively recent development of wire wound prestressed concrete pipe greater internal pressures can be carried and greater resistance to external loading afforded.

The wire wound prestressed concrete pressure pipe has advantages of economy of steel and quality of concrete to satisfy engineering designs for high pressure heads. The magnitude of internal pressure resulting from hydrostatic head and external loadings, to be resisted by the pipe will govern the design for quantity of steel wire and the amount of prestress necessary to use in the wrapping.

Refer to Figure 5;

f_s stress in the steel, p.s.i.

f'_s stress in the steel due to internal pressure, p.s.i.

f_c stress in the concrete, p.s.i.

f'_c reduction in concrete prestress, p.s.i.

A_s area of steel, square inches.

A_c area of concrete, square inches.

r internal radius of pipe, inches.

t wall or shell thickness, inches

P internal pressure, p.s.i.

E_s modulus of elasticity of steel, p.s.i.

E_c modulus of elasticity of concrete, p.s.i.

n E_s/E_c.

d_s deformation in steel, inches per inch.

d_c deformation in concrete, inches per inch.

1. When the shell is wound with a wire in tension there is produced;

(a) a tensile stress in the wire, f_s.

(b) a compressive stress in the concrete, f_c.

In this derivation the average stress in the steel and in the concrete is used. The stress in the steel must equal the stress in the concrete to maintain equilibrium, hence;

$$f_c A_c = f_s A_s$$

For 1 inch length of pipe $Ac = t$, then:

$$fc = \frac{fs\ As}{t}$$

2. When the prestressed pipe is subjected to internal pressure there is produced:

(a) An increase in the steel prestress

(b) A decrease in the concrete pre compression.

Hence, for equilibrium:

$$Pr = f'_s\ As + f'_c\ Ac$$

$$d_s = \frac{f's}{E_s} \quad \text{and } dc = \frac{f'c}{Ec}$$

But $ds = dc$; therefore $\frac{f's}{E_s} = \frac{f'c}{E_c}$

Whence $f'_s = f'_c \frac{E_s}{E_c}$; but $\frac{E_s}{E_c} = n$

Hence $f'_s = f'_c n$ which is the increase in steel stress accompanying the decrease in concrete pre compression due to internal pressure. Since the concrete is assumed to take no tensile stress the limit of f'_c is the magnitude of the concrete prestress f_c, therefore:

$$Pr = nf_c A_s + f_c A_c ; \quad A_c = t \text{ for 1" length}$$

$$P = f_c \frac{(nA_s + t)}{r}$$

$$f_c = \frac{Pr}{nA_s + t}$$

The curves shown in Figure 1 taken from an article by Ray E. Crepps in the A.C.I. Proceedings illustrate a simple means of design for any one particular pipe which is subjected to a varying hydrostatic head. By plotting both design equations on the same set of axis a convenient and rapid solution of the equations is available. An example to illustrate the use of these curves is as follows:

Suppose a pressure pipe with a head of 250 p.s.i. is to be used. Entering the curves with 250 p.s.i. and projecting horizontally to the internal pressure curve, then vertically to the steel area curve, read directly the area of steel necessary per foot of length of pipe; in this case 0.04 square inches per foot.

WATER TANKS AND STAND PIPES

The next logical step in the use of prestressing would be in the construction of a water tank or stand pipe. The theory of design here presented was taken from an article published in the A.C.I. publication "R/C" which was written by Lt. Comd'r John L. Mason (CEC) USNR. Various other articles on the subject were studied in conjunction with the design presented in this thesis; however it is the opinion of the authors that Lt. Comd'r Mason has presented the most concise and logical development of the necessary design formulae. Only the most pertinent formulae will be presented here. For a more complete development, and for examples of uses of these formulae reference to Lt. Comd'r. Mason's article or the design herein presented is suggested.

Consider a ring of concrete with a band of steel laid snugly around its exterior surface but not stressed. By means of a turnbuckle or other mechanical device keep shortening the band until it is stressed to f_{si}, the initial steel stress. The total initial force on the steel band will be $A_s f_{si}$. This force must, of course, be in equilibrium with the forces in the concrete ring, hence:

$$A_s f_{si} = A_c f_{ci} \quad \text{or} \quad f_{ci} = p f_{si} \quad \text{where } p = \frac{A_s}{A_c}$$

Suppose now that the prestressed ring is subjected to an internal hydrostatic pressure of q per unit of length and the radius of the ring is R; then the ring tension, T in the combined concrete and steel section is:

$T = q R$. The tensile stresses due to ring tension are;

for the concrete $= \dfrac{T}{(Ac + nAs)}$

for the steel $= \dfrac{nT}{(Ac + nAs)}$

where $n = \dfrac{Es}{Ec}$.

The combined final stresses are:

for the concrete $fc = -pfsi + \dfrac{T}{Ac\,(1 + np)}$

for the steel $fs = fsi + \dfrac{T}{As\,(1 + np)}$

As seen from the above equations the initial compressive stress in the concrete was reduced and the initial stress in the steel was increased. For complete insurance against cracking the usual procedure is to consider the combined stress in the concrete equal to zero. Setting the equation for final concrete stress equal to zero and solving for the initial steel stress we have;

$$fsi = \frac{T}{As\,(1 + np)}$$

When $f_c = 0$ the total ring tension must be taken by the steel, hence $T = A_s f_s$. Therefore for the initial steel stress we have;

$$fsi = \frac{fs}{1 + np}$$

This is, however, based on the assumption that the concrete behaves as a perfectly elastic material with no shrinkage. This assumption is, of course, false and a correction to the initial stress must be taken into account due to actual shrinkage. The steel bands must be given an actual stress such that;

$$fsl = \frac{fs + CEs}{1 + np}$$ where C = shrinkage coefficient.

(For derivation of this latter equation see "Circular Concrete Tank Without Prestressing", A.C. I. "R/C" No. 4).

Lt. Comd'r. Mason indicates that, from his experience, he has found the value of C to be approximately 0.0002 for tanks above ground level. He also indicates that the value of p should be limited to approximately 0.02.

From the maximum percentage of band steel an equation for minimum wall thickness t can be developed in terms of ring tension T:

$$p = \frac{A_s}{12t} = \frac{T/fs}{12t}$$

$$t = \frac{T}{12pf_s}$$

Hence t = the minimum wall thickness that can be used. If the steel stress is constant throughout the entire height of wall then the thickness t must remain constant. By using the relation that $A_s = \frac{T}{fs}$ the area of steel necessary at any point in the wall can be determined.

STRAIGHT LINE MEMBERS

A more recent application of the theory of prestressing, and one that is coming into prominence, is the application of the theory to straight line members such as beams, columns, and girders.

The theory applied to these elementary components may be carried into the construction of larger structures. It has been claimed by advocates of the theory that when applied to bridges there results a structure which gives excellent resistance to the effect of concentrated wheel loads, impact and vibrations. Long span bridges of the simple beam and cantilever type can be designed with a relatively small depth-span ratio combined with small deflections (according to Schorer).

The theory herein presented is taken from an article by Herman Schorer in the Journal of the American Concrete Institute. As previously stated in the Introduction there is a loss in the initial steel prestress due to the following causes:

(1) Shrinkage of the concrete.

(2) Plastic flow of the concrete.

(3) Elastic deformation of the concrete.

The total change in the initial stress in the steel may be given approximately by the following empirical formula:

$$\text{Change in } f_s = 15{,}000 + 15\, f_c \qquad \text{(a)}$$

Hence the final effective steel stress is:

$$f_s = f_{so} - \text{change in } f_s$$

The effective stress can be determined by means of successive approximations. For this purpose the entire original prestressed force P_o is first assumed to be acting, whereby an approximate value of the concrete prestress is obtained. The corresponding steel stress reduction is then determined from the above equation. This approximation is then used to obtain a second approximation for the effective prestress force, and this procedure is repeated until the corrections become negligible.

A derivation of the design formulae for beams and girders follows;

Notation;

Change in f_s = total steel stress reduction.

f_c = effective concrete stress.

f_{so} = original steel stress.

P_o = original steel prestress force.

f_{cop} = effective concrete stress at the c.g.c. due to prestress force, P.

M_p = internal moment, due to prestress force, P.

f_{c1p} = extreme concrete fiber stress due to prestress force, P.

f_{c2p} = extreme concrete fiber stress due to prestress force, P.

s_1 = S_{c1}/A.

s_2 = S_{c2}/A.

k_1 = f_{c1p}/f_{cop}, stress ratio.

k_2 = f_{c2p}/f_{cop}, stress ratio.

f_{cop} = effective concrete stress at the c.g.s. due to prestress force, P.

c_1 = extreme fiber distance from c.g.c.

c_2 = extreme fiber distance from c.g.c.

K_0 = f_{c2p}/f_{cop}, stress ratio.

M_0 = dead load moment.

a = distance between center gravity concrete and neutral axis.

A = area of transformed section.

A_s = steel area.

f'_{c1m} = concrete fiber stress change due to external moment, M.

f'_{c2m} = concrete fiber stress change due to external moment, M.

f'_{csm} = concrete stress change at the c.g.s. due to external moment, M.

f_{sm} = steel stress change due to live load moment, M.

S_{c1} = section modulus of fiber c_1, referred to c.g.c.

S_{c2} = section modulus of fiber c_2, referred to c.g.c.

M = live load moment.

Figure 6 shows a beam section with an effective prestress force, P acting at the distance, e, from the c.g.c. The average effective concrete stress is,

$$f_{cop} = \frac{P}{A_c} \tag{1}$$

The eccentric application causes an internal moment,

$$M_p = eP \tag{2}$$

The extreme fiber stresses are,

$$f_{c1p} = \frac{P}{A_c} + \frac{eP}{S_{c1}} \tag{3}$$

$$f_{c2p} = \frac{P}{A_c} - \frac{eP}{S_{c2}} \tag{4}$$

By designating,

$$a_1 = \frac{S_{c1}}{A_c} \tag{5}$$

And,

$$s_2 = \frac{Ic2}{Ac} \qquad (6)$$

The fiber stresses are then determined by means of stress ratios;

$$k_1 = \frac{fc1p}{fcop} = \left(1 + \frac{e}{s_1}\right) \qquad (7)$$

and,

$$k_2 = \frac{fc2p}{fcop} = \left(1 - \frac{e}{s_2}\right) \qquad (8)$$

The concrete stress at the c.g.s. is,

$$fcsp = fc1p - (fc1p - fc2p)\frac{(C_1 - e)}{C_1 + C_2} \qquad (9)$$

which after transformation gives,

$$fcsp = \frac{(1 + e^2 Ac)}{Ic} \qquad (10)$$

or by introducing the stress ratio,

$$ks = \frac{fcsp}{fcop} = \frac{Is}{Ic} \qquad (11)$$

the effective steel stress is,

$$fsp = \frac{P}{As} \qquad (12)$$

The originally required steel stress can now be determined by substituting fcsp in equation (a),

Change in fs = fcsp + 15 fc

The designer is now confronted with the opposite problem; that is, the determination of the effective stress due to the release of the original prestress force, P_o, also the simultaneous influence of dead loads.

If the dead load moment, M_o, is smaller than the counteracting prestress moment, equation (2), the release of the prestress force will cause the beam to lift off the supporting forms. In this case the dead load acts simultaneously with the prestress load and the steel stress loss is determined by the combined concrete stress at the c.g.c. The concrete stress, f_{ceMo}, due to dead load moment is,

$$f_{ceMo} = \frac{Mo}{Se} \qquad (13)$$

The effective stresses are preferably determined by the method of successive approximations. For this purpose the known value of P_o, instead of P is at first substituted in equation (1), which results in an approximate stress f_{cep} as derived from equation (11). This stress is combined with the given dead load stress, equation (13). The resulting stress, substituted in change of prestress equation, gives an approximation for the total steel stress reduction; therefore a correction for the effective prestress force.

The effective fiber stresses are now obtainable from equations (7) and (8) by means of the final value of P, as substituted in equation (1). In the case of top and bottom reinforcing the corrections should be made simultaneously, although the two prestress forces can be treated independently as a first approximation.

The stress changes due to live loads must now be considered. The stress changes due to live loads are based on the transformed section, assuming that volume changes can be considered as completed. The transformed area, A, is based on the entire concrete section,

$$A = Ac + nAs \qquad (14)$$

With reference to Figure 6 the location of the neutral axis is given by the relation,

$$a = \frac{enAs}{A} \qquad (15)$$

$$I = I_c + a^2 Ac + (e - a)^2 nA_s \qquad (16)$$

or,

$$I = Ic + eaAc \qquad (17)$$

With $c'1 = c_1 - a$ the section modulus for the bottom fiber becomes,

$$S'_{c1} = \frac{I}{c'1} \qquad (18)$$

For the top fiber, with $c'_2 = c_2 + a$,

$$S'_{c2} = \frac{I}{c'2} \qquad (19)$$

For the c.g.s. with $e' = e - a$,

$$S'e = \frac{I}{e'} \qquad (20)$$

The concrete stress changes produced by the live load moment, M, then are,

$$f'_{c1m} = \frac{M}{S'c1} \qquad (21)$$

$$f'_{c2m} = \frac{M}{S'c2}$$

$$f'cem = \frac{M}{S'e}$$

and the steel stress change, from equation $f_s = nfc$ becomes,

$$fsm = n\, f'cem.$$

COMPARATIVE DESIGN

The purpose of this comparative design is to arrive at a measure of the relative economy of materials required for a structure designed first, by conventional reinforced concrete design procedure and second, by using the theories of prestressed structures. For the purpose of an overall economic study such a comparison of designs should, of course, take into account the relative costs of fabrication and the labor charges. A determination of these latter factors is, however, beyond the scope of this study; hence the comparison will be based solely upon the quantity of materials required in each case.

For the purpose as outlined above the structure to be compared will be a 500,000 gallon circular water storage tank. The comparison will be extended in the case of the tank cover to include a flat slab cover in one case and a segmental dome cover in the other. These types were chosen as being best suited to the individual tank designs.

Notation

A_c = area of concrete section, square inches.

A_s = area of steel in tension, square inches.

b = width of rectangular beam, inches.

C = shrinkage coefficient of concrete.

d = effective depth, inches.

D = diameter of tank, feet.

E_c = modulus of elasticity of concrete in compression, p.s.i.

E_s = modulus of elasticity of steel, p.s.i.

f_c = compressive unit stress in extreme fiber of concrete, p.s.i.
f'_c = ultimate compressive strength of concrete, p.s.i.- at age of 28 days.
f_s = tensile unit stress in longitudinal reinforcement, p.s.i.
f_{si} = initial stress in steel, p.s.i.
f_{ci} = initial stress in concrete, p.s.i.
H = height of wall, feet.
H_1 = hoop stress in dome at point 1, kips per foot.
I = moment of inertia of horizontal cross section of tank, inch units.
I_s = moment of inertia of reinforcement about neutral axis, inch units.
j = ratio of distance between centroid of compression and center of gravity of tensile reinforcement to depth "d".
k = ratio of distance between the compressive face of the beam and the neutral axis to the depth "d".
M = moment due to dead and live loads, foot pounds.
n = ratio of modulus of elasticity of steel to that of concrete.
p = ratio of area of tensile reinforcement to the effective area of concrete in beams and slabs.
r = radius of sphere, feet.
R = radius of tank, feet.
S_c = compression in edge member of lantern opening of dome, kips.
S_1 = ring tension in edge member of dome, kips.
t = thickness of beam or wall, inches.
T = hoop stress, pounds.
T_1 = meridional stress in dome, kips per foot.

v = unit shear in concrete, p.s.i.

V = total shear, pounds.

w = equivalent water pressure, pounds per cubic foot.

w_1 = combined dead and live load on dome, kips per square foot.

W_1 = weight of dome, kips.

ϕ_0 = angle subtended by axis of dome and edge of lantern opening, degrees.

ϕ_1 = angle subtended by axis of dome and support edge, degrees.

Δ = deflection, inches.

DESIGN DATA

REINFORCED CONCRETE TANK:

(1) Circular tank resting on ground surface.

(2) Allowable soil bearing - 3000 psf.

(3) Allowable stress in circumferential wall steel - 12,000 psi. (This low allowable stress is chosen to prevent excessive elongation of the steel in case the concrete cracks).

(4) Allowable stress in all other steel - 18,000 psi.

(5) f'_c - 3000 psi.

(6) E_s - 30,000,000 psi.

(7) Ultimate concrete tensile strength = 250 psi.

(8) Required tank height (effective) - 20 feet.

(9) Assume base is fixed.

(10) Specifications: A.C.I. 318-41 and Bu. Y.&D. Specs. -3YB.

(11) Unit weight of concrete - 150 pounds per cubic foot.

(12) Live Load on cover - 25 psf.

(13) Wind pressure - 5/8 x 30 psf of vertical projection.

PRESTRESSED CONCRETE TANK:

(1) Same diameter and effective height as R.C. tank. Same soil conditions.

(2) Allowable stress in prestressing steel - 22,500 psi. prior to prestressing.

(3) Allowable stress in reinforcing steel - 18,000 psi.

(4) f'_c - 3000 psi.

(5) E_s- 30,000,000 psi.

(6) Ultimate concrete tensile strength- 250 psi.

(7) Assume base is fixed.

(8) Unit weight of concrete- 150 pounds per cubic foot.

(9) Specifications: A.C.I. 318-41

(10) Live Load on cover- 30 psf.

(11) Collar load on dome- 15 kips.

(12) Wind stress - 5/8 x 50 psf of vertical projection

REINFORCED DESIGN

DIMENSIONS:

Vol. = 500,000 gal. = 54,000 cu. ft.

Diameter = 64' -0" Height = 20'

THICKNESS OF WALL:

Pressure at bottom of wall = 62.5 x 20 = 1250 p.s.f.

Pressure 1 ft. up from bottom = 62.5 x 19 = 1187 p.s.f.

Ave. for 1 ft. section = $\frac{1}{2}$ (1250 + 1187) = 1218 psf

T for 1 ft. section = $1218 \times \frac{64}{2}$ = 39,000#

To allow for shrinkage in the concrete assume coefficient of shrinkage C = 0.0004

$$p = \frac{f_c}{nE_cC + f_s - f_cn} =$$

$$= \frac{250}{10(3,000,000)(.0004) + 12000 - 250 \times 10}$$

$$= 0.0116$$

$$t = \frac{T}{12pf_s} = \frac{39,000}{12 \times .0116 \times 12,000} = 23.4"$$

Use t = 24"

Let thickness at top = 8"

AREA OF STEEL:

Assume concrete cracks and steel takes all of load

$A_s = \frac{39000}{12000} = 3.25$ sq. in. per ft. of Ht.

Per inch of height;

$A_s = \frac{3.25}{12} = 0.271$ sq. in. per inch.

A_s each side $= \frac{0.271}{2} = 0.136$

Try 7/8" ϕ

Spacing $= \frac{0.6013}{0.136} = 4.42"$ USE 4"

Change steel spacing at 5 ft. intervals.

At 5 ft. above base;

$T = 62.5 \times 15 \times \frac{64}{2} = 30\,000$ # per ft. of Ht.

$A_s = \frac{30,000}{12,000} = 2.50$ sq. in. per ft.

$A_s = \frac{2.50}{12} = 0.208$ sq. in. per inch.

For two rows $A_s = 0.104$ in. per in.

For 7/8 ϕ

Spacing $- = \frac{0.6013}{0.104} = 5.78$ USE $5\frac{1}{2}$ inches

At 10 ft. above base

$T = 62.5 \times 10 \times \frac{64}{2} = 20,000$ # per ft. of Ht.

$A_s = \frac{20000}{12000} = 1.667$ sq. in. per ft.

$A_s = 0.139$ sq. in. per in.

for two rows As = 0.0695 in. per in.

For 7/8 ø

$$\text{Spacing} = \frac{0.6013}{0.0695} = 8.65 \quad \text{USE } 8\tfrac{1}{2} \text{ in.}$$

At 15 ft. above base;

$$T = 62.5 \times 5 \times \frac{64}{2} = 10{,}000 \;\#$$

$$A_s = \frac{10{,}000}{12{,}000} = 0.833 \text{ in. per ft.}$$

As = 0.0695 inches per in.

For two rows As = 0.0348 in. per in.

For 5/8 ø

$$\text{Spacing} = \frac{0.31}{0.0348} = 8.9 \text{ in.} \quad \text{USE } 8\tfrac{1}{2} \text{ in.}$$

MOMENT IN WALL DUE TO FIXED BASE:

$$\text{Max. deflection} = n f_s \frac{D}{2E_s} = \frac{2500 \times 64}{2 \times 30{,}000{,}000}$$

$$= 0.00267 \text{ ft.}$$

$$= 0.032 \text{ in.}$$

Allowing 2" Cover for steel.

$A_c = 24 \times 12 = 288$ sq. in.

Take $A_s = 0.01 A_c = 0.01 \times 288 = 2.88$ sq. in.

$t = 24"$

$p = 62.5 \times 20 = 1250$ p.s.f. $= 104$ #/in. of Ht.

$I_s = 2.88 \times 10 \times 10 = 288$ inch units

$I_c = \frac{1}{12} \times 12 \times 24^3 + 9 \times 288 = 13{,}800 + 2590$

$Ic = 16{,}390$ inch units

Let h_1 = height above base to which cantilever action extends.

$$\text{Deflection} = \frac{1}{80} \frac{(p^1 h_1^{\ 4})}{EI}$$

or $h_1 = 124.5 \ (5.05)^{\frac{1}{4}} = 186.5$ in.

$h_1 = 15.55$ ft.

MOMENT AT BASE:

$$M_1 = \frac{104 \times (186.5)^2}{8} = 451{,}000 \text{ in \#.}$$

Taking $d = 20"$

$j = 0.857$

$k = 0.429$

$$A_s f_s = F = \frac{451{,}000}{0.857 \times 20} = 26{,}400 \ \#$$

Assume f_s = 18,000 p.s.i.

A_s = $\frac{25400}{18000}$ = 1.465 sq. inches for 1ft. section.

As = 0.122 sq. inches / inch

For 7/8 ∅

Spacing = $\frac{0.6013}{0.122}$ ≈ 4.93 USE 4½ in.

POINT OF CONTRAFLEXURE:

Point of C.F. = 0.37 h_1

= 0.37 x 15.53 = 5.75 ft. up from base.

Extend rods to 7 ft. above base

COMPRESSIVE STRESS IN CONCRETE:

$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 451{,}000}{0.429 \times 0.857 \times 12 \times 400}$

= 511 p.s.i. @ Bott

MAXIMUM MOMENT ABOVE POINT OF CF.

$M = 1/3\ M_1$

$A_s = 1/3\ A_{s1} = \frac{0.122}{3}$ = 0.041 sq. inches/inch

for 1/2" in ∅

Spacing = $\frac{0.20}{0.041}$ = 4.88 USE 4½"

EXTEND TO 15½ Ft. ABOVE BASE.

Extend every fourth rod inside & outside to top for support of curcumferential steel and to use as temperature steel.

SHEAR AT TOP OF BASE:

$V = \frac{1250}{2}$ x 15.53 = 9700 #/Ft. of width.

$v = \frac{V}{bjd} = \frac{9700}{12 \times 0.857 \times 20}$ = 47 psi.
Allowable = 60 psi

DESIGN FOR CONES:

Assuming simple supports at wall centerline and center of supporting column. USE Span of 32' - 0"

Snow and Live load = 25 psf

Wt. of concrete = 150 #/cu. ft.

Total load = 175 #/ cu. ft.

Reaction @ Wall $= 175 \times \frac{32}{2} \times \frac{2}{3} \times \frac{32}{32} = 1865^{\#}$

Reaction @ Column $= 175 \times \frac{32}{2} \times \frac{1}{3} \times \frac{32}{32} = 934^{\#}$

Maximum moment @ 0.577 L from Small end.

Max. Moment = 0.1283 WL

$= 0.1283 \times 175 \times \frac{32}{2} \times 32 = 11{,}500$ Ft. #

WIDTH OF SECTION AT MAX. MOMENT

Section of max. moment = 0.577 x 32 = 18.5' from small end.

Width $= \frac{18.5}{32} \times 12 = 6.94"$

$f_c = \frac{2M}{kjbd^2}$

$d^2 = \frac{2M}{f_c kjb} = \frac{2 \times 11{,}500 \times 12}{1350 \times 0.429 \times 0.857 \times 6.94}$

d = 8.95" USED = 9"

USE 3" Cover Total D = 12"

$A_s = \frac{M}{f_s jd} = \frac{11{,}500 \times 12}{18{,}000 \times 0.857 \times 9} = 0.995$ sq. in.

A_s/ inch $= \frac{0.995}{6.94} = 0.1432$ sq. in.

USE 7/8" ∅

Spacing $= \frac{0.6013}{0.1432} = 4.2$ in. USE 4.25"

Check rod spacing at wall

$$\frac{18.5}{32} = \frac{4.25}{x} \quad \text{Let } x = \text{spacing @ wall}$$

$$x = \frac{4.25 \times 32}{18.5} = 7.35 \quad \text{Use } 7$$

SHEAR AT WALL:

R = 1665 #

$$v = \frac{V}{bjd} = \frac{1665}{12 \times 0.887 \times 9} = 201 \text{ psi.}$$

WALL BEARING CHECK:

$$\text{Bearing} = \frac{1665}{8 \times 12} = 19.4 \text{ psi.}$$

Shear and bearing at column end of cover will be checked after design of column.

DESIGN OF BASE:

Assume;

footing width = 6' - 0"

Allowable soil bearing pressure = 3000 psf.

Depth of footing = 20 inches.

LOADS:

Load on top of wall per foot of wall	=	1665 #
Wt. of wall per foot	=	4000 #
Wt. of base	=	1490 #
Total		7355 #/ft. of wall

$$\text{Soil bearing due to dead load} = \frac{7355}{6} = 1226 \text{ psf}$$

Total load due to water on base:

$$\pi \times 32 \times 32 \times 1250 = 4,020,000 \#$$

Area of base:

$$\pi \times 36 \times 36 = 4070 \text{ sq. ft.}$$

Soil bearing due to water:

$$\frac{4,020,000}{4070} = 987 \text{ psf}$$

Total soil bearing pressure at foot of wall:

$$987 + 1226 = 2213 \text{ psf.}$$

3000 psf allowed.

Considering the part of the foundation that extends beyond the wall to act as a cantilever and taking moments about the face of the wall we have -

$$M = 12 \times (2213) \times 6 = 53,100 \text{ in. lbs.}$$

$$A_s = \frac{M}{f_s jd} = \frac{53,100}{18000 \times 0.857 \times 17}$$

$$A_s = 0.202 \text{ sq. in. per foot}$$

Using $\frac{1}{2}$" ⌀ Area = 0.20

$$\text{Spacing} = \frac{0.20}{0.202} = .99 \text{ Ft.}$$

USE 12" Spacing for 1/2" ⌀

SHEAR CHECK AT FACE OF WALL:

$$v = \frac{4420}{12 \times 0.857 \times 17} = 25.3 \text{ psi}$$

Allowable = 60 psi.

CHECK FOR WIND LOAD:

Assume 50 #/ sq. ft.

Load = 5/8 x 50 x 64 = 2000 # /Ft. of Ht.

Total = 20 x 2000 = 40,000 #

Mom. = 40000 x 10 = 4000,000 Ft. #

MOMENT OF INERTIA OF CROSS SECTION:

$$I = \frac{\pi}{4}\left(\frac{^{4}}{33.33} - \frac{^{4}}{32}\right) = 145{,}000 \frac{^{4}}{\text{Ft.}}$$

STRESS DUE TO SIDE LOAD:

$$f = \frac{MC}{I} = \frac{400{,}000 \times 33.33}{145{,}500} = 91.5 \text{ \#/Ft. of wall}$$

This additional loading would not change dimensions of footing.

COLUMN DESIGN:

Roof slab reaction on column = 188,000 #

Estimated weights:

Capital =	4530 #
Drop panel =	3200 #
Column =	5560 #
Total	13,290

Therefore load on base slab = 13,290 + 188,000

= 201,290

For 20" column:

Load on concrete = 212,000 # from ACI Design Hand book

Theoretical load on the steel = 0

Use 3/8" hoops on 12" Centers and

Vertical Steel = 6 - 5/8 ∅

Size of column and steel, though not required to carry load were chosen to lend rigidity which is the important factor.

Soil bearing at base of column.

Use 10' -0" Dia. base slab.

Area = 78.6 sq. ft.

Wt. of Base 78.6 x 1 x 150 = 11,800 #

Total wt. on soil = 11,800 + 201,290 = 213,090

$$\text{Stress} = \frac{213,090}{78.6} = 2,710 \text{ p.s.f.}$$

CHECK OF SHEAR AND BEARING OF [illegible] ON COLUMN CAPITAL:

Assume bearing at 2 ft. from center of column.

Bearing area = 11.25 sq. in.

Reaction on column = 934 #

$$\text{Bearing pressure} = \frac{934}{11.25} = 83.4 \text{ psi}$$

SHEAR CHECK:

At a distance d = t - 1½ from edge of column. Capital b = 1.45"

d = 10.5"

$$v = \frac{V}{bjd} = \frac{934}{1.45 \times 0.857 \times 10.5} = 71.5 \text{ psi.}$$

Allowable = 90 psi.

DESIGN OF FLOOR SLAB:

Except for the cantilever section of the base no moment occurs in the base slab, Assuming uniform settlement of the soil. Therefore the slab will not be designed to resist moment. However, to assume the settlement over the entire area to be uniform might lead to

DESIGN OF FLOOR SLAB - Continued :

dangerous cracking of the floor of the tank. To prevent this a relatively thin slab will be laid. The flexibility of the thin slab will be such that it can conform to any slight differential settlement without producing cracks.

(See drawing for dimensions and steel in base slab)

PRESTRESSED DESIGN

WALL THICKNESS & STEEL AREA:

Pressure at bottom of wall

$T = wHR = 62.5 \times 20 \times 32 = 40{,}000$ #/Ft.

Assuming the Concrete to Take Zero Stress when Tank is Full:

$$A_s = \frac{T}{f_s} = \frac{40{,}000}{22{,}500} = 1.78 \text{ sq. in./Ft.}$$

(See Fig. 2 for steel spacings)

Adding 1 foot at bottom of tank for construction joint and using 4 bands @ 3" spacing, total number of bands used will be 41 3/4" ϕ.

INITIAL STRESS:

$$f_{si} = \frac{f_s + C' E_s}{1 + np} = \frac{22{,}500 + 0.0002 \times 30 \times 10{,}000{,}000}{1.14}$$

$= 25{,}000$ psi allowable initial steel prestress.

WALL THICKNESS:

Current practice limits percentage of steel in bands to 2% of concrete area; hence

$$t = \frac{1.78}{0.02 \times 12} = 7.34" \quad \text{Use } 8"$$

$$\text{Max. percentage of steel} = \frac{1.76}{8 \times 12} = .01833$$

CHECK ON INITIAL CONCRETE STRESS:

Since $f_{ci} = -pf_{si}$.

$f_{ci} = -.01833 \times 25{,}000 = 458$ psi Allowable $= 1350$ psi

Minimum steel percentage at top of wall

$$\frac{0.33}{8 \times 12} = 0.00344 \text{ Therefore}$$

f_{ci} (Min) $= -.00344 \times 25{,}000 = 86$ psi.

TENSION BAND LENGTH:

Radius to center line of bands

Radius $= 32 + \frac{8 + .375}{12} = 32.692$ ft.

Circumference $= 2\pi \times 32.692 = 206.0$ ft.

USE 4 bars per band length 51.5' per bar.

Total number of bars $= 4 \times 41 = 164$ @ 51.5' and 164 turn buckles

CHECK FOR TENSILE STRESS DUE TO PRESTRESSING BANDS:

When the bottom bands are prestressed to a ring tension of

$1.76 \times 25{,}000 = 44{,}000$ #/ft.

they will exert a radial pressure on the wall equal to their ring tension divided by their radius,

$\frac{44{,}000}{32.69} = 1347$ #/ft.

This corresponds to a water pressure of

$\frac{1347}{20} = 67.3$ #/ft.

Using moment coefficient tables given in "MODERN DEVELOPMENTS IN REINFORCED CONCRETE".

Entering table with:

$\frac{H^2}{Dt} = \frac{20 \times 20}{64 \times 0.667} = 9.36$

And $WH^3 = 67.3 \times \overline{20}^3 = 538{,}000$ ft. #/ft.

Point	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	1.0H
Coeff. from Table VIII	0.0	0.0	0.0	-.0002	-.0001	.0004	.0014	.0029	.0047	.0049	0
Moment	0	0	0	-107.6	-53.8	134.5	154	1560	2530	2640	0

See Moment curve Fig. 3. (appended)

From moment diagram max. moment is,

2650 ft. # @ 17.5' from top.

Vertical steel necessary to resist moment

$$A_s = \frac{M}{f_s j d} = \frac{2650 \times 12}{18000 \times 0.857 \times 6} = .344 \text{ sq. in./ft.}$$

USE 1/2" ⌀ @ 7" spacing.

Compressive concrete stress due to moment,

$$f_c = \frac{2650 \times 12 \times 4}{\frac{1}{12} \times 12 \times 8^3} = 244 \text{ p.s.i.} \qquad \text{Allowable} - 250 \text{ p.s.i.}$$

DESIGN OF WALL FOOTING

Load on top of wall per ft.	=	1590#
Wt. of wall per ft.	=	2100#
Assumed depth of footing 12" wt.	=	600#
Total		4290#

Assume width of footing = 4' -0"

$$\text{Soil bearing} = \frac{4290}{4} = 1071 \text{ p.s.f.}$$

Total load due to water = 4,020,000#

Area of base 3420 sq. ft.

$$\text{Soil bearing due to water} = \frac{4{,}020{,}000}{3420} = 1175 \text{ p.s.f.}$$

Total soil bearing = 1071 + 1175 = 2246 p.s.f.

Allowable 3000 p.s.f.

Using a footing section as shown in drawing of prestressed tank critical moment will occur in section marked (a)

Moment at point (a)

$$M = 3690 \times 4 = 14,760 \text{ in.}^{\#}$$

Check for fo.

$$fo = \frac{2 \times 14,760}{0.429 \times 0.857 \times 10 \times 10 \times 12} = 65 \text{ psi}$$

Allowable = 250 psi.

$$A_s = \frac{M}{f_s jd} = \frac{14,760}{18,000 \times .857 \times 10} = .096 \text{ sq. in./ft.}$$

Using 1/2" ∅ Area 0.20

$$\text{Spacing} = \frac{0.20}{.096} = 2.09 \text{ ft.} \quad \text{USE } 12"$$

Check Shear at critical Section.

$$v = \frac{V}{bjd} = \frac{3690}{12 \times 0.857 \times 10} = 36.0 \text{ psi}$$

WIND STRESS:

Moment = 400,000 ft. # (from previous design)

$$I = \frac{\pi}{4}\left(32.67^4 - 32^4\right) = 47,200 \text{ ft. units}$$

$$f = \frac{400,000 \times 32.67}{47,200} = 277 \ \#/_{\text{ft. of wall}}$$

Total soil bearing = 2525 p.s.f.

Therefore wind stress does not change dimensions of footing.

DOME DESIGN FOR PRESTRESSED TANK:

Refer to Figure 4 for general information.

Radius of sphere:

$$r^2 = 32 \times 32 + (r - 8)^2$$

$$r = 67.8 \text{ ft.}$$

Assume 5" thickness:

$$w1 = 0.0625 + 0.030 = 0.0925 \text{ kips/sq. ft.}$$

$$\text{Sin. } \phi_o = \frac{3}{67.8} = 0.0442; \quad \text{Cos } \phi_o = 0.995$$

$$\text{sin } \phi_1 = \frac{32}{67.8} = 0.472; \quad \text{Sin}^2 \phi_1 = 0.222$$

$$\text{Cos } \phi_1 = 0.882$$

$$\phi 1 = 28^\circ$$

$$W1 = 2\pi \times 67.8 \times 67.8 \times 0.0925 \, (0.995 - 0.882)$$

$$= 320 \text{ kips.}$$

$$T1 = \frac{320}{2\pi \times 67.8 \times 0.222} = 3.39 \text{ kips/ ft.}$$

$$= \frac{3390}{5 \times 12} = 56.5 \text{ p.s.i.}$$

$$H1 = -3.39 + 0.0925 \times 67.8 \times 0.882 = 2.11 \text{ kips/ ft.}$$

$$= 35.2 \text{ p.s.i.}$$

$$S1 = \frac{320 \times 0.882}{2\pi \times 0.471} = 95.3 \text{ kips. ring tension in edge member.}$$

$$So = \frac{15 \times 0.995}{2\pi \times 0.0442} = 53.7 \text{ kips compression in edge member of lantern.}$$

EDGE MEMBER FOR DOME:

$$\text{Bearing on wall} = \frac{320{,}000 =}{201.5 \times 12 \times 8} = 16.6 \text{ p.s.i.}$$

Allowable = 750 p.s.i. in bearing.

Ring tension = 95.5 kips; $A_s = \frac{95.5}{20} = 4.76$ sq. in. USE 6 - 1" ∅.

Shear on 5" section; $v = \frac{320,000}{201.5 \times 12 \times 5} = 26.6$ p.s.i. (O.K.)

Temperature steel - Use 1/4" ∅ @ 12" both ways.

LANTERN COPING:

Compression in edge member = 53.7 kips

Allowable on concrete = 1360 p.s.i.

$A = \frac{53,700}{1360} = 39.8$ sq. in.

Use 6" x 7" coping.

COMPARISON OF RELATIVE ECONOMY

As previously stated the comparison of the preceding designs will be based solely upon the quantity of materials (steel and concrete) used in each case, no attempt being made to evaluate fabrication costs and labor charges due to the many variable and unknown factors affecting the latter costs.

The total quantity of steel rods required in the case of the reinforced concrete tank is 82,277 pounds, and the corresponding total volume of concrete is 446 cubic yards. In the case of the prestressed concrete tank the total quantity of steel rods required is 27,734 pounds and the total volume of concrete is 318 cubic yards.

For these figures the difference in quantity of materials and the per cent saving gained by using prestressed construction are as follows;

	R.C. Design	Prestressed Design	Difference	% Savings
Total Steel	82,277#	27,734#	54,543#	66.3%
Total Concrete	446 cu. yd.	318 cu. yd.	128 cu. yd.	28.6%

These figures include the additional saving gained by using the dome cover with the prestressed construction in preference to the flat slab cover of the reinforced concrete construction. The quantities of materials required for these covers alone (but including the center supporting column in the case of the flat slab) are;

	R.C. Design	Prestressed Design	Difference	% Savings
Total Steel	19,020 #	1,110 #	17,910 #	94%
Total Concrete	129 cu. yd.	51 cu. yd.	78 cu. yd.	60.5%

Then, assuming the same cover (flat slab) in both designs, the quantity of materials in each case would represent a true comparison of the relative economy of materials for the two types of construction. These figures are:

	R.C. Design	Prestressed Design	Difference	Savings
Total Steel	82,277 #	45,644 #	36,633 #	44.5%
Total Concrete	446 cu. yd.	396 cu. yd.	50 cu. yd.	11.2%

Conclusions

An indication of the advantages to be gained by using prestressed concrete construction is given by the figures representing the quantity of materials required for this type of construction compared to the quantity required for conventional construction. It is realised that a comparison made solely on this basis is not indicative of the overall costs in each case, but a complete study of this matter is beyond the scope of this thesis as actual construction costs were not available to the authors.

On the other hand, the figures presented for comparison do not reveal all of the advantages gained by using prestressed construction in this case. For the prestressed tank cracklessness is guaranteed and this quality is, of course, of primary importance in a pressure vessel. To obtain a comparable degree of security in this respect in the case of the reinforced concrete tank it was considered advisable to reduce the allowable tensile stress in the circumferential steel to 12,000 p.s.i. or about 67 per cent of the usual value for tensile reinforcement. For the corresponding steel in the prestressed tank it was possible to utilise the full allowable stress, higher strength steel being used economically in this case.

It should also be pointed out that the lower total weight of the structure and hence the lower soil bearing pressures (for the same size footings) might be the controlling factor in a location of low soil bearing.

The free base which is well suited to the prestressed construction obviates the necessity of providing for moment in the circumferential footing at the base of the wall.

These factors would, in some cases, counter-balance the additional cost of construction for a prestressed tank and thus lend more reality to the figures presented for comparison.

In the process of design of the prestressed tank the authors attempted to design a flat slab cover using prestressed beam theory but this was found to be impractical from a construction standpoint due to the requirement of a constant percentage of steel throughout the cover to maintain economy of design. The difficulty encountered in carrying out this requirement would present itself in the fabrication stage. It appears to be impractical to cut off any of the rods at the interior of the span of the slab, to anchor these rods at the interior point, and to apply the necessary force for prestressing either before or after setting of the concrete. Further study of this subject will be required to devise a means of overcoming these difficulties in order that the prestressing theories may be applied to members whose cross sectional area varies with length.

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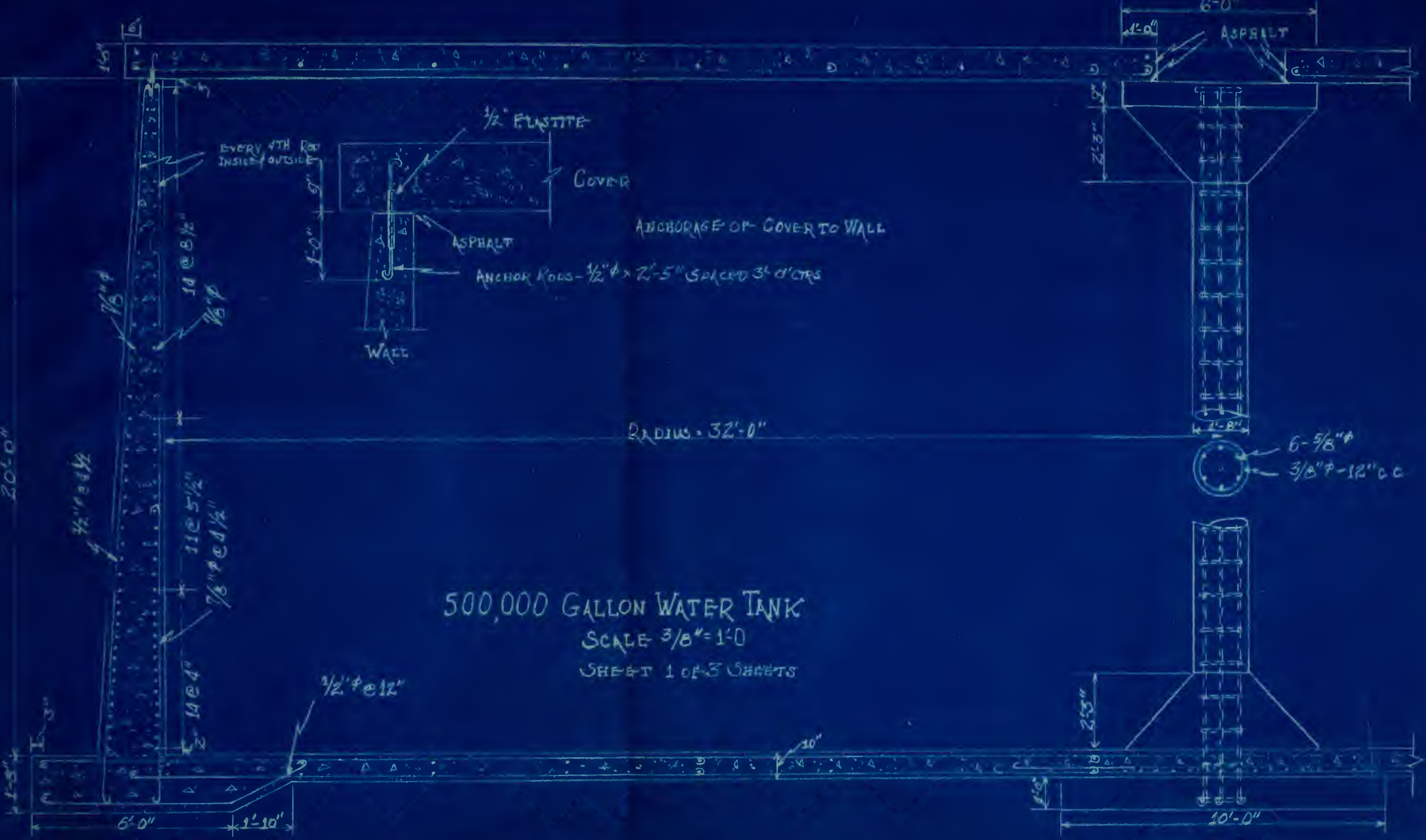
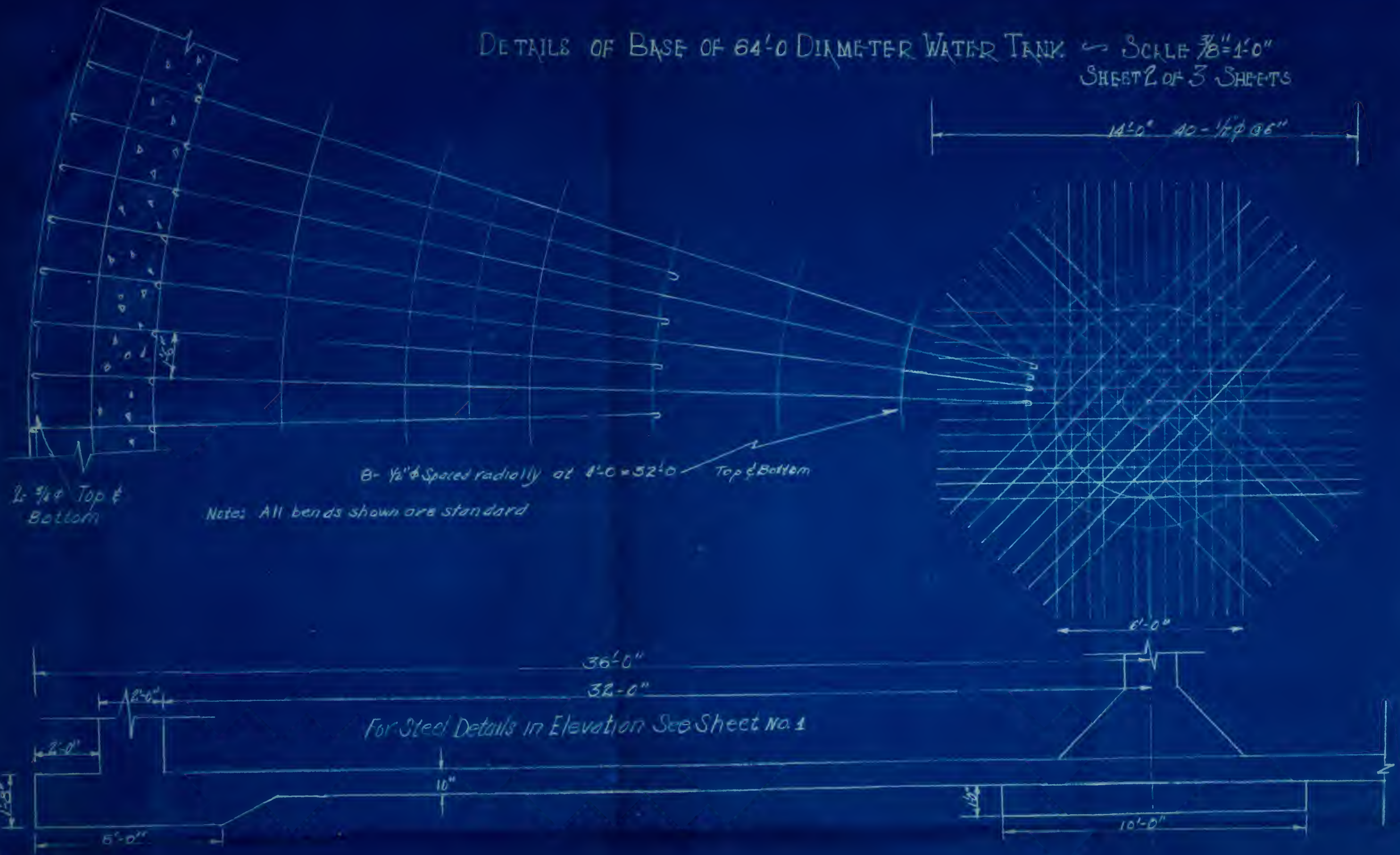
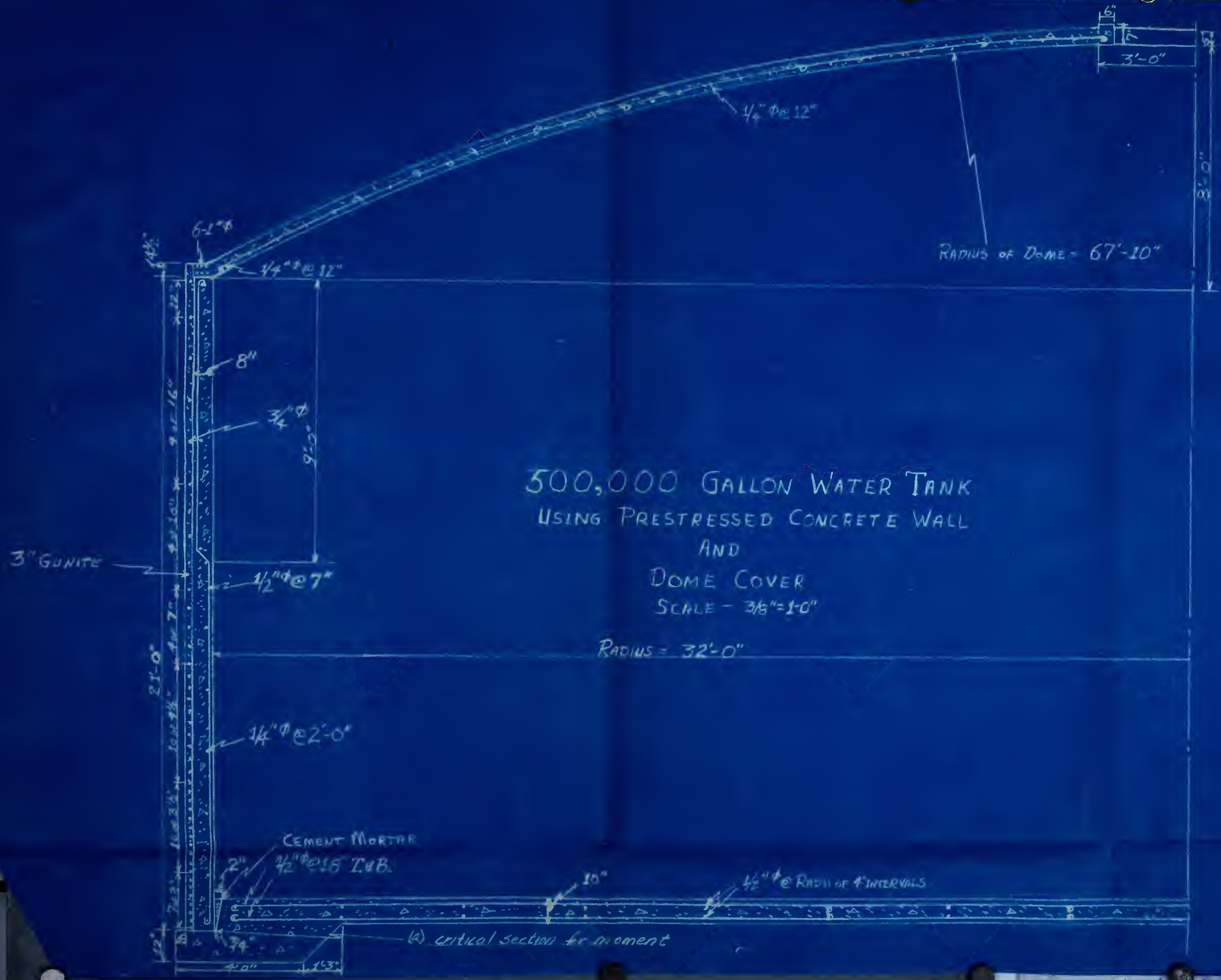


500,000 GALLON WATER TANK
SCALE 3/8" = 1'-0
SHEET 1 OF 3 SHEETS

6-0"
1'-0"
ASPHALT
6"
1'-6"
3"
1/2" ELASTITE
EVERY 4TH ROD INSIDE & OUTSIDE
COVER
9"
1'-0"
ASPHALT
ANCHORAGE OF COVER TO WALL
ANCHOR RODS - 1/2"φ × 2'-5" SPACED 3' O'CTRS
WALL
14 @ 8½"
7/8"φ
7/8"φ
20'-0"
RADIUS = 32'-0"
2'-3"
1'-8"
6-5/8"φ
3/8"φ-12" O.C.
11 @ 5½"
7/8"φ @ 4½"
½"φ @ 4½"
14 @ 4"
2"
3"
1/2"φ @ 12"
10"
2'-3"
1'-3"
6'-0"
1'-10"
1'-0"
10'-0"





500,000 Gallon Water Tank
Using Prestressed Concrete Wall
And
Dome Cover
Scale – 3/8" = 1'-0"

6"
3'-0"
8'-0"
1/4" Φ @ 12"
Radius of Dome = 67'-10"
6-1" Φ
1/4" Φ @ 12"
8"
3/4" Φ
9'-0"
3" Gunite
1/2" Φ @ 7"
Radius = 32'-0"
21'-0"
9 @ 16"
1/4" Φ @ 2'-0"
Cement Mortar
2"
1/2" Φ @ 16 T & B.
10"
1/2" Φ @ Radii of 4' intervals
(a) critical section for moment
3/4"
4'-0"
1'-3"
12"

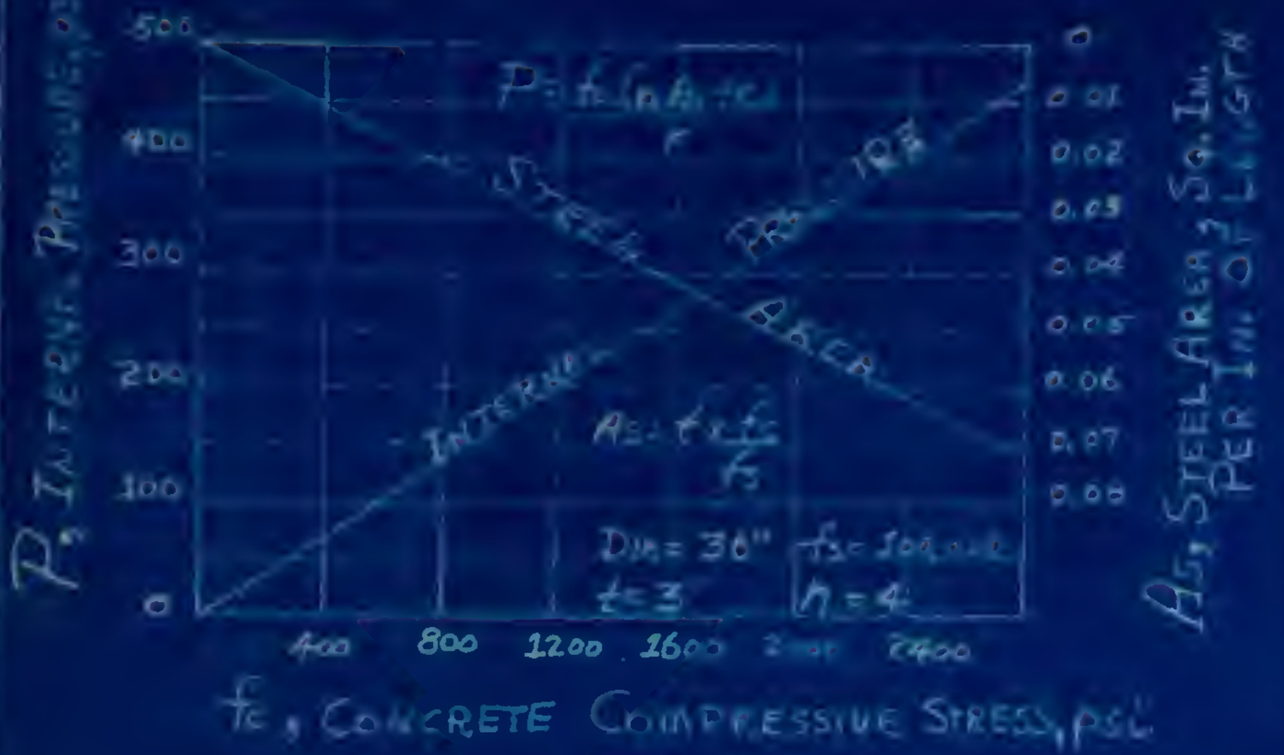


Fig. 1

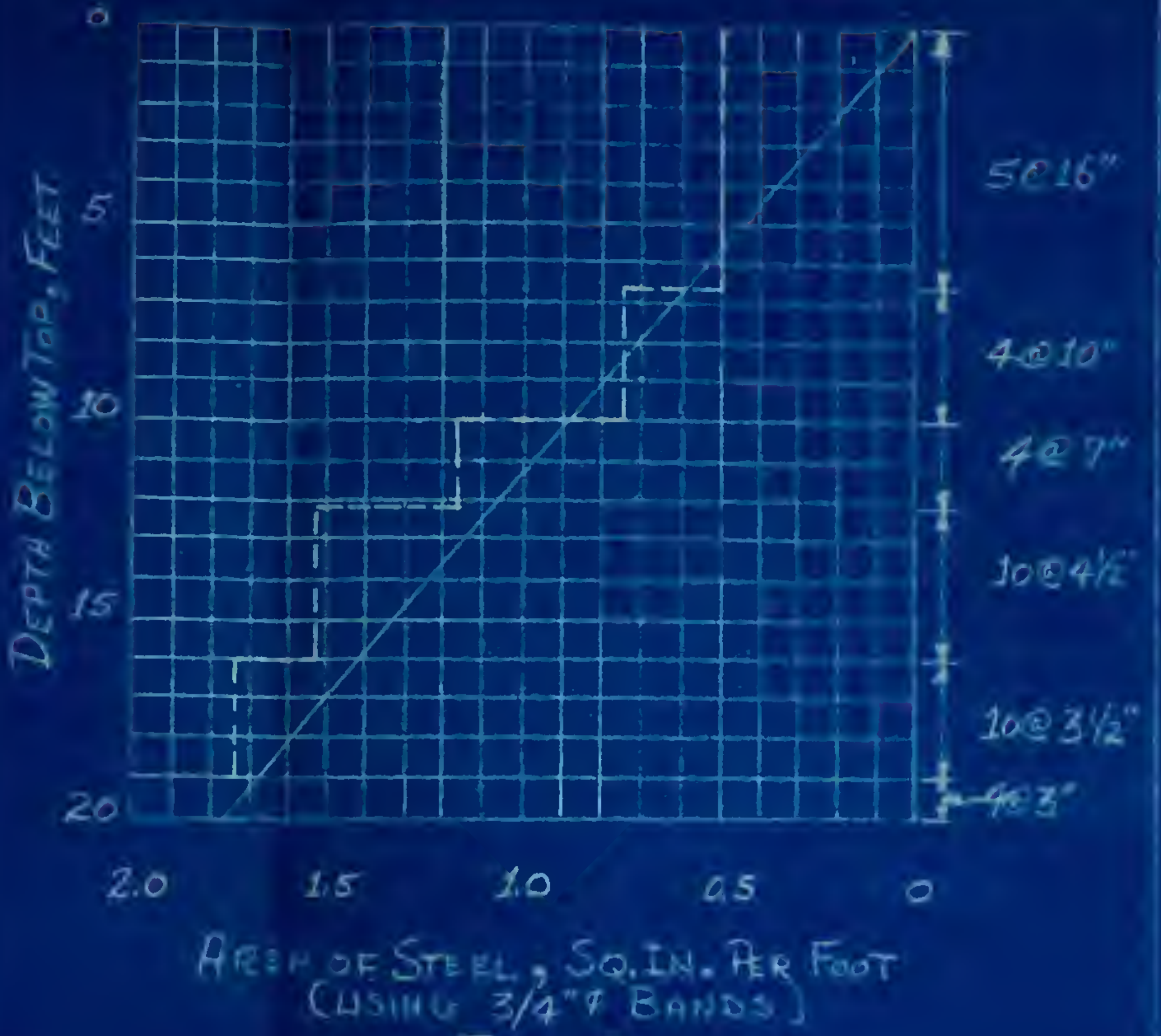


Fig. 2

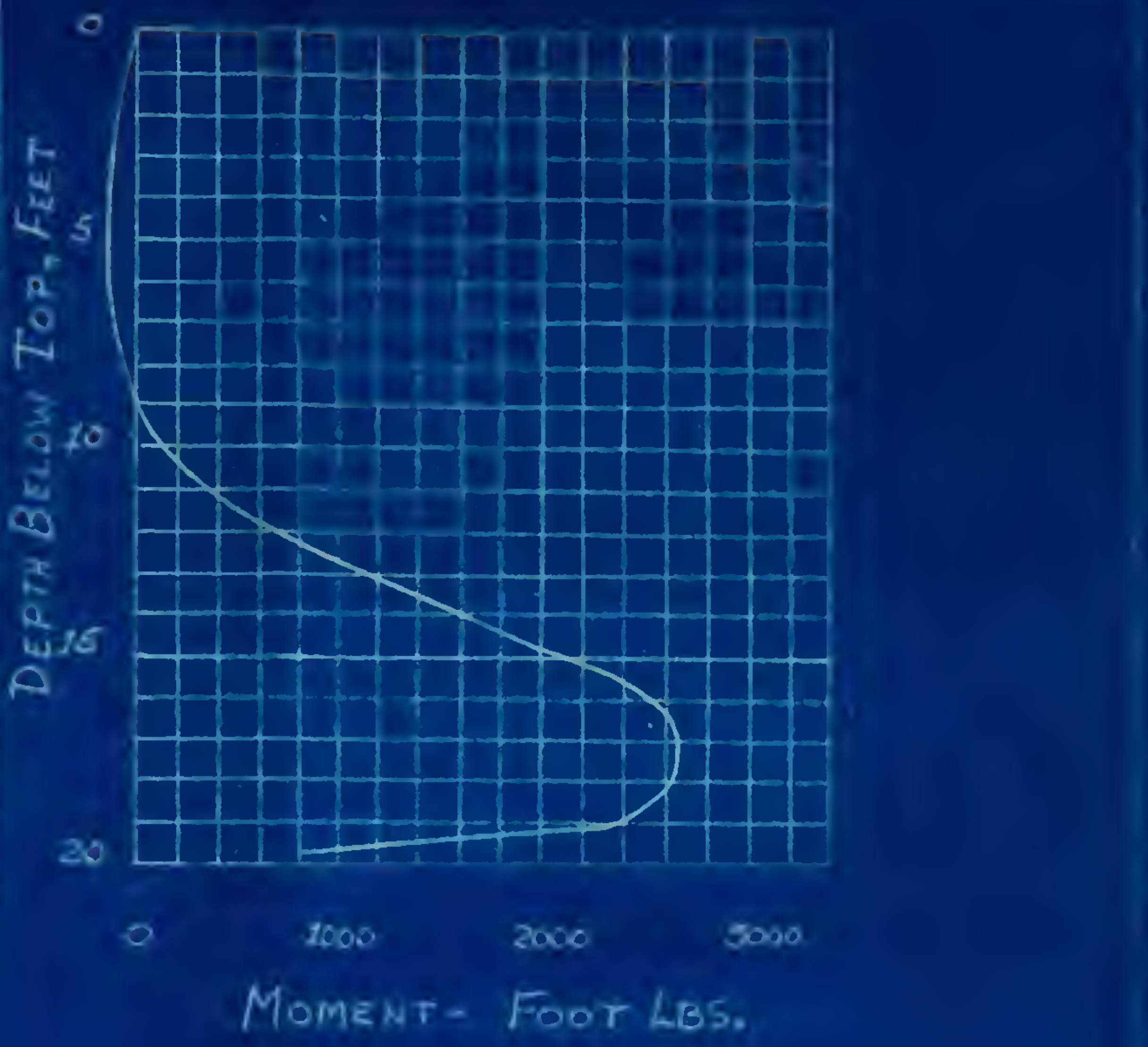


Fig. 3

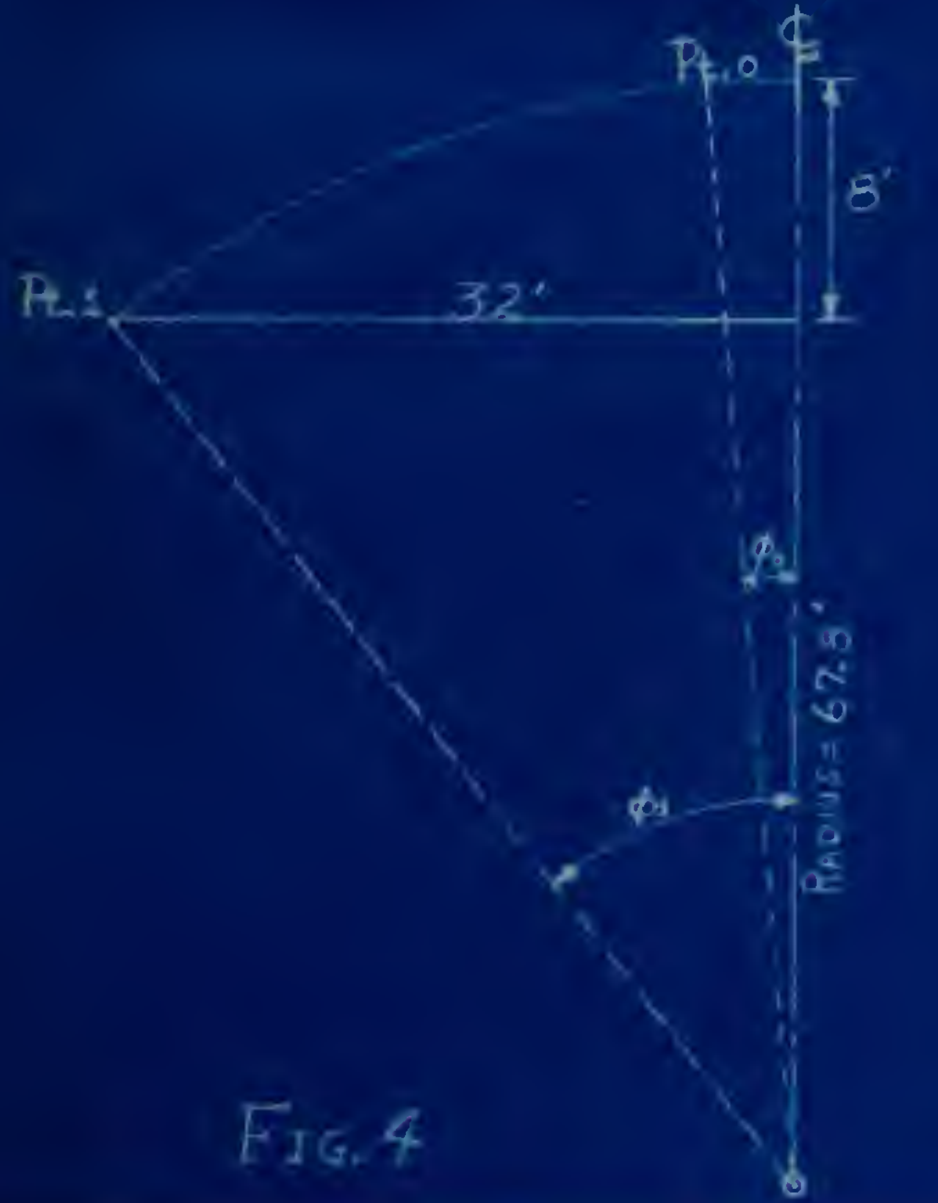

Pt. 0
℄
8'
Pt. 1
32'
ϕ_2
ϕ_1
Radius = 67.5'

Fig. 4

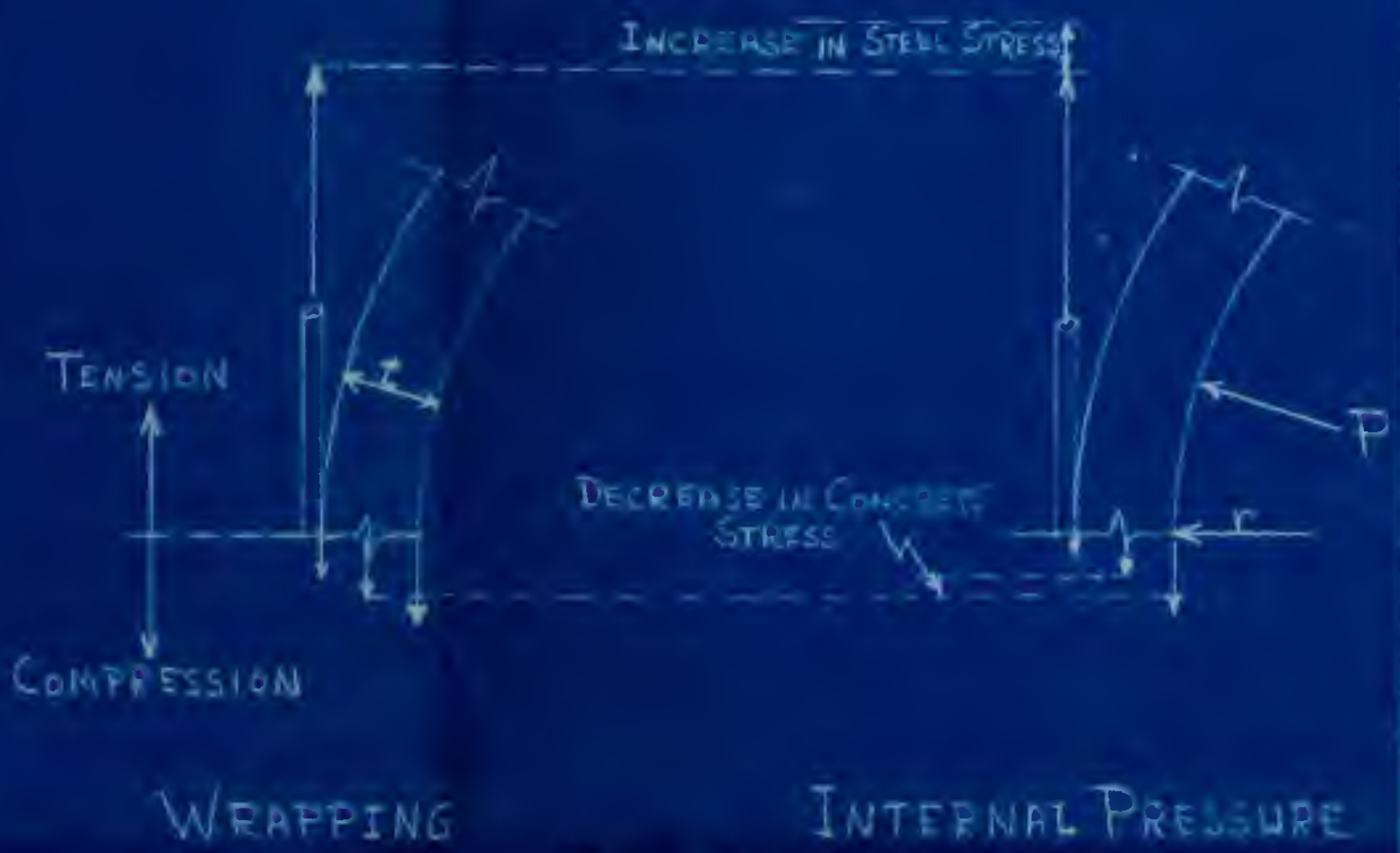
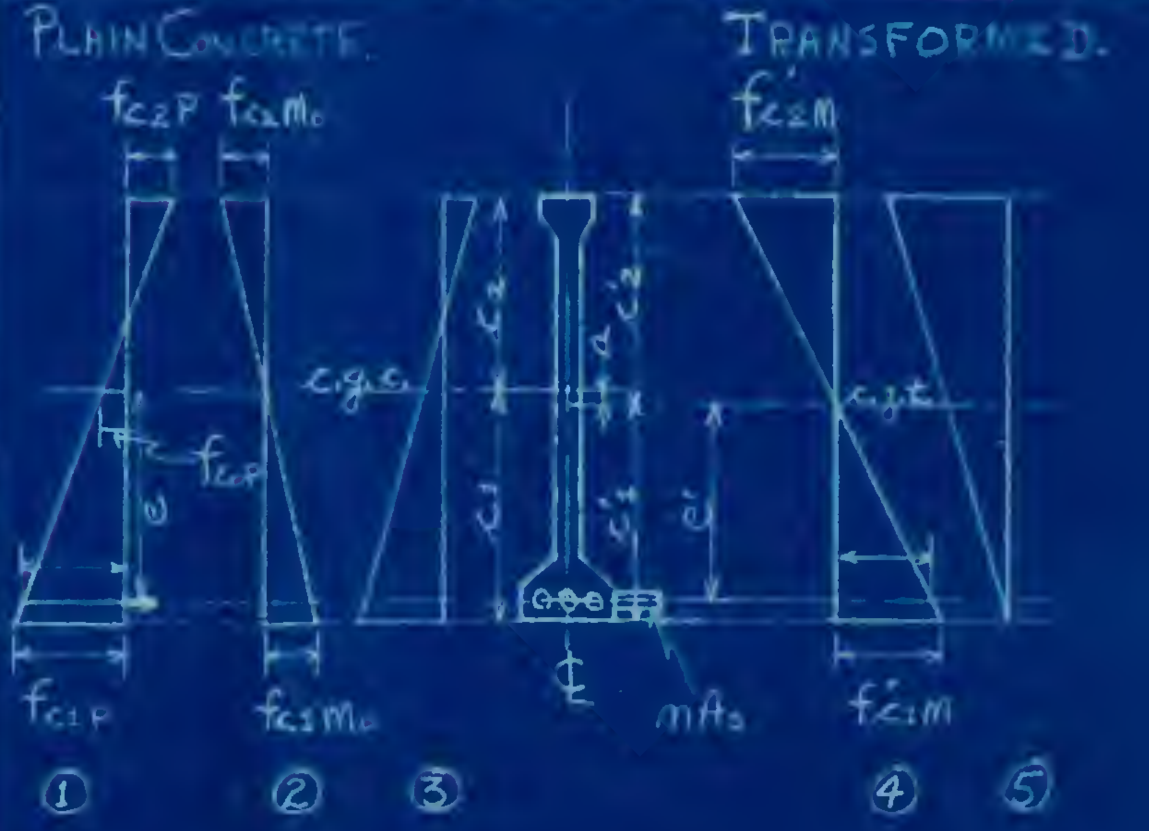

Increase in Steel Stress
Tension
Compression
t
Decrease in Concrete Stress
P
r
Wrapping
Internal Pressure

Fig. 5


Plain Concrete.
Transformed.
f_{c2P}
f_{c2M_0}
f'_{c2M}
c.g.c.
c.g.t.
f_{cP}
e
c_2
c_1
c'_2
c'_1
d
e'
f_{c1P}
f_{c1M_0}
℄
nA_s
f'_{c1M}
①
②
③
④
⑤

① = Prestress Load.
② = Dead Load.
③ = ① + ② Combined.
④ = Live Load.
⑤ = ③ + ④ Combined.

Fig. 6.

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